Conference Paper, Published Version

Ligier, Pierre-Louis

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Two-dimensional modelling of flow conditions generated by piled piers and turbulence-based erosion risk assessment

Pierre-Louis Ligier
Hydropower and Dams Department
Sweco Energuide AB
Stockholm, Sweden
pierre-louis.ligier@sweco.se

***Abstract*—A hydraulic study has been performed in the Gavleån River in Sweden, to assess the impacts of an architectural project composed of piled piers to be built into the riverbed in the city centre of Gävle. The aim of the study was to assess the impacts in terms of high water levels, change in flow conditions and erosion risk. Hydraulic modelling was performed with a two-dimensional model, TELEMAC-2D. The article presents a description of the study area, of the architectural project and of the hydraulic model. A description of the methodology used to model the piers is given, with some of the piers having their deck being submerged during high flows, thus inducing an increased flow resistance. The impacts of the project are an increase of flow velocities in the centre of the river due to flow contraction between piers and quays and an increase of turbulence below the piers generated by the piles. The erosion risk has been assessed with a turbulence-based approach, in which the bottom shear stress is calculated from the turbulence parameters given by the k-ε turbulence model instead of from the local depth-averaged flow velocity and bed friction parameters.**

I. INTRODUCTION

A hydraulic study has been performed in the Gavleån River in Sweden, to assess the impacts of the "Å-rummet" architectural project composed of piled piers to be built into the riverbed in the city center of Gävle. The aim of the study was to assess the impacts in terms of high water levels, change in flow conditions and erosion risk. Firstly, the article gives a presentation of the study area and of the Å-rummet project. In a second part, the two-dimensional hydraulic model developed, using the software TELEMAC-2D, is detailed and the method used to account for flow resistance generated by the piers is described. The project impacts on water levels and flow velocities are then analyzed. Finally, the method used to perform the erosion risk assessment, which is based on turbulence parameters, is presented and results are discussed. The work presented in this article has been performed as part of a consulting assignment during approximatively 70 hours.

II. STUDY AREA AND PROJECT DESCRIPTION

The city of Gävle is located approximately 170 km north of Stockholm, Sweden, where the Gavleån River is released into the Baltic Sea, see Fig. 1. The last 2.5 km of the Gavleån River are located in the city centre of Gävle where the river

Figure 1: Geographical location of the city of Gävle, Sweden (red dot).

Figure 2: Overview of the city of Gävle and loation of the Å-rummet project in the Gavleån River.

banks are artificial and composed of quays and harbour piers. The city of Gävle has started a recreation project called "Å-rummet" which aims at making the centre of the city and the promenade along the river more attractive by building piers within the riverbed. The location of the Å-rummet project within the city centre is depicted in Fig. 2. Artistic illustrations of the planned piers are presented in Fig. 3.

The Å-rummet project consists of building nine piers and two pedestrian bridges on a total length of about 500 m at the upstream part of the artificialized reach, see Fig. 4. The pedestrian bridges will not interfere with the river for any discharge and have therefore no significance regarding hydraulic impacts. The nine piers will be divided into three types with i) pier 1 composed of a concrete slabs founded on piles, ii) piers composed of steel structures founded on piles (pier 3, 6, 7, 8 and 9) and iii) suspended piers composed of steel structures without contact with the riverbed (pier 2, 4 and 5). The piers can be temporary submerged depending on river discharges and downstream sea levels. The three pier types are illustrated in Fig. 5. Pier 1 has a total length of approx. 190 m and a width of approx. 10 m. Its lower face has varying elevations, ranging from 0.2 to 1.0 m above mean sea level in the downstream direction. The other piers are much smaller with horizontal dimensions of approx. 5 x 10 m and their lower faces are located at different elevations, the lowest being 0.9 m above mean sea level.

The piles have a diameter of 0.3 m including an ice protection layer. The structure of the piers' lower face is composed of either concrete (pier 1, height 0.4 m) or steel beams (other piers, height 0.195 m) that will generate friction and turbulence when submerged. The expected impacts induced by the piers are additional head losses and increased turbulence generated by the piles and by the piers' lower face roughness leading to a redistribution of the velocities across the river.

Figure 3: Artistic illustration of the Å-rummet project. Pier 1 seen from downstream.

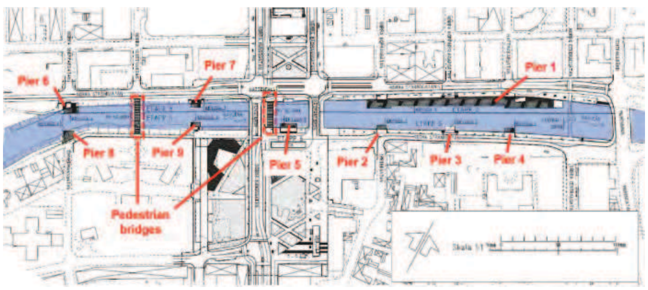

Figure 4: Overview of the Å-rummet project. Flow direction: left to right.

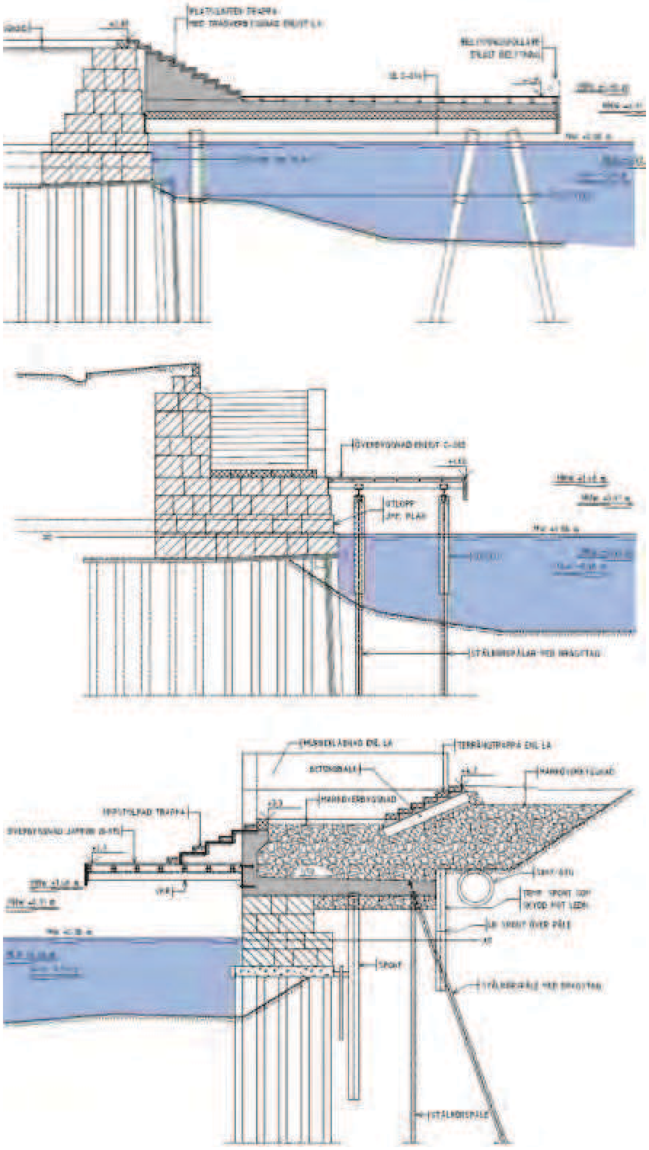

Figure 5: Three different pier types. Top: concrete slab founded on piles (pier 1). Middle: steel structures founded on piles (pier 3, 6, 7, 8 and 9). Bottom: suspended steel structures without contact with the riverbed (pier 2, 4 and 5).

III. Presentation of the Hydraulic Model

A. *Type of hydraulic model*

The flow conditions in the estuary of Gavleån River can be complex due to the discharge of fresh water in a stratified water body (Baltic Sea). However, in this part of the Baltic Sea the salinity is low with values in the range of 6 g/l with a weak stratification. The water depths in Gavleån River are rather small (maximum 3 m along the Å-rummet project) which means that during flood events the flow conditions are likely to be close to two-dimensional.

The Å-rummet project adds complexity to the flow conditions especially when piers start to be submerged. However, the study presented in this article has been performed with a two-dimensional hydraulic model (TELEMAC-2D version 7.1). This assumption is reasonable as i) not all the piers are submerged, ii) the area of the submerged piers is small (excepted for pier 1) and iii) the absence of calibration data and detailed riverbed material data prevented from using a three-dimensional model.

B. Mesh and bathymetry

The computational meshes covers approx. 2.5 km of Gavleån River from upstream of the Å-rummet project down to the river mouth in the harbour area (see Fig. 6). Two meshes were created, one for the current state geometry and one for the project geometry. Mesh size is approx. 1 m along the Å-rummet project and approx. 3 m downstream. The mesh has been refined around existing bridge piles with a mesh size of approx. 0.5 m and pier piles of the Å-rummet project has been discretized with a mesh size of approx. 0.1 m (see Fig. 7). It has been assumed that all piles a purely vertical. Current state and project meshes are composed of approx. 77,000 and 111,500 triangular elements, respectively.

A digital elevation model has been created using available echo sounding survey, bridge drawings and LIDAR data for land in the upstream bend (see Fig. 8). Water depths in Gavleån River related to mean sea level are comprised between 1.5 and 3 m along the Å-rummet project and progressively increase in the downstream direction to reach approx. 4 to 5 m below the railway and highway bridges and

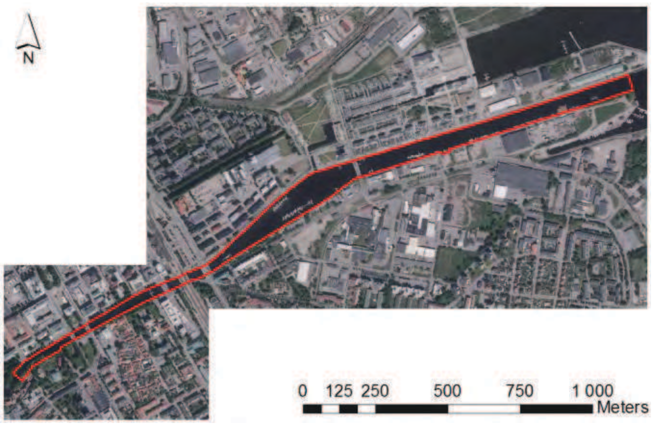

Figure 6: Model domain.

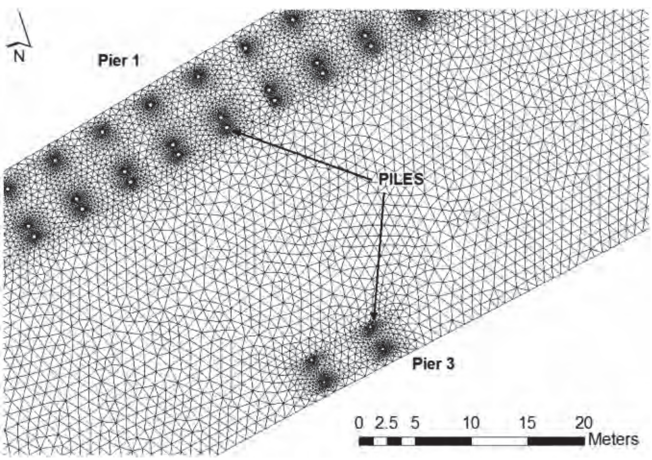

Figure 7: Detailed view of the mesh for the project geometry.

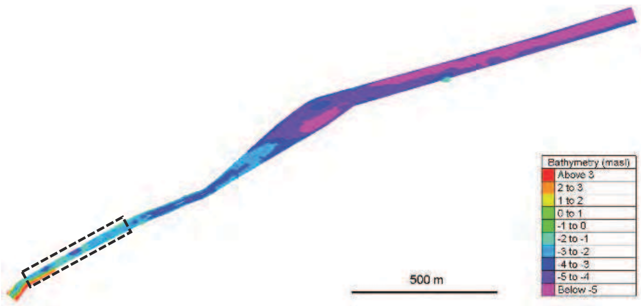

Figure 8: Digital elevation model. Project location is indicated by the dashed polygon.

approx. 4 to 6 m in the lower part of the reach just upstream the river mouth.

C. Numerical parameters

Bottom friction has been modelled using Strickler's friction law with a friction coefficient of 25 $m^{1/3}/s$ based on available information on river geometry and riverbed material and existing hydraulic studies in the same river reach. It is recalled that no calibration data was available meaning that the absolute results are somewhat uncertain but the relative differences on both water levels and flow velocities between the current state and project geometries can be assessed with a reasonable level of uncertainty. Simulations were performed with a time-step of 0.1 second until steady state conditions were reached. Turbulence was modelled with a k-ε turbulence model which offers the advantages of providing the local depth-averaged turbulent kinematic energy used in the erosion risk assessment as well as being a well-established model for river flow applications.

D. Modelling of flow resistance induced by the piers

The flow resistance induced by the piers is generated by the piles and by the submerged decks. The flow resistance generated by the piles is modelled directly by the hydraulic model as each pile is included in the model geometry. The submerged decks generate two types of flow resistance with i) additional friction generated by the irregular profile of the lower face (beams) and ii) flow contraction once the deck structure itself is submerged. Friction term is expressed as shear stress, see Eq. 1 [1].

$$\vec{\tau} = -\frac{1}{2} \cdot \rho \cdot C_f \cdot U \cdot \vec{U} \tag{1}$$

Where $\vec{\tau}$ is the shear stress vector (N/m^2), ρ the water density (kg/m^3), C_f the quadratic friction coefficient (-), U and $\vec{U}$ the depth-averaged velocity component and vector, respectively (m/s). The quadratic friction coefficient is dimensionless and can be expressed by different friction law such as Strickler and Nikuradse, see Eq. 2 and Eq. 3.

$$C_{f,St} = \frac{2 \cdot g}{St^2 \cdot h^{1/3}} \tag{2}$$

$$C_{f,ks} = 2 \cdot \left[\frac{\kappa}{ln\left(\frac{11 \cdot h}{k_s}\right)} \right]^2 \tag{3}$$

Where St is the Strickler friction coefficient (m$^{1/3}$/s), h the water depth (m), g the gravitational acceleration (9.81 m/s^2), κ the von Karman constant (0.4) and k_s the equivalent sand roughness coefficient (m).

Bottom friction is modelled using Eq. 2 as detailed in section III.C above. The additional friction induced by the submerged decks has been modelled with Eq. 3 in which the equivalent sand roughness is taken as the submerged height of each beam, based on local water level, see Eq. 4.

$$k_s = MIN\big(H_B; (WL - LF_B)\big) \tag{4}$$

Where H_B is the total beam height (0.4 m for pier 1, 0.195 m for other piers), WL the local water level (masl) and LF_B the elevation of the beam's lower face (masl). The LF_B parameter values have been assigned to mesh nodes using the PRIVATE VARIABLES procedure making it possible to compute the actual equivalent sand roughness coefficient at each computational node.

Friction terms have then been modelled with $C_f = C_{f,St}$ at nodes affected by bottom friction only and with $C_f = C_{f,St} + C_{f,ks}$ at nodes affected by both bottom and pier friction. Flow contraction effects have not been modelled. The absence of calibration data and detailed riverbed material survey prevented from using the Nikuradse friction law to model bottom friction. This approach would have been preferable in order to ensure that the total quadratic friction coefficient could be based on the same friction law. Nonetheless, the method used in this study can be considered as acceptable being given the uncertainties and simplifications at play (calibration data, material, 2D model) in relation to the study's scope.

E. Boundary conditions and simulated cases

Simulations have been performed for a combination of two design flows prescribed at the upstream boundary (Q_{50} = 168 m^3/s and Q_{100} = 210 m^3/s) and three sea levels prescribed at the downstream boundary (mean sea level +0.06 m, average of yearly highest sea levels +0.91 m and average of yearly lowest sea levels -0.49 m).

IV. Project impacts on water levels and flow velocities

A. Water levels

The project impacts on water levels have been estimated by comparing longitudinal profiles extracted in the river axis for the two simulated geometries for each flow case. Such a comparison is presented in Fig. 9 for the 100-year flood combined with a mean sea level. Results show that for this particular flow case pier 1 is submerged on nearly all its length (approx. 180 m) with a maximal submergence of approx. 0.8 m at the upstream end. This generates head losses in the reach in which water levels increase between 0.09 m and 0.25 m upstream of pier 1. For this particular flow case, pier 6, 7 and 9 are also submerged while pier 3 and 5 have only a

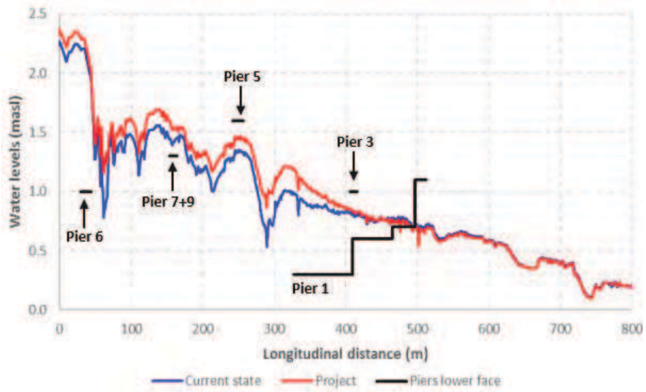

Figure 9: Water level evolutions. Longitudinal profile in river axis. 100-year flood with mean sea level.

0.15 m freeboard. Pier 2, 4 and 8 are located above elevation +3.0 m and are thereby not affecting flow conditions. Project impacts for other flow cases are varying within ± 0.05 m from the impacts presented above.

B. Flow velocities

As for water levels, impacts are presented for the 100-year flood combined with a mean sea level flow case. Analysis of flow velocities in current state shows that strong velocities occur at the upstream end of the Å-rummet project location, just downstream of pier 6 due to a cross section contraction at the end of the bend (approx. 5 m/s). Flow velocities along the project location are mainly varying between 2.0 and 3.5 m/s. Along pier 1 flow velocities are somewhat lower, especially in the downstream part, ranging from 1.0 to 2.5 m/s.

The velocity evolutions generated by the project are strongest in the reach along pier 1 and 3 with i) a decrease of velocities under pier 1 up to -1.5 m/s along the outer pile row exposed to flow, ii) a decrease of velocities up to -0.3 m/s in the wake of pier 3 and iii) an increase of flow velocities in the middle of the river up to 0.5 m/s due to the flow contraction generated by pier 1 and 3 (see Fig. 10). Detailed analysis of velocity changes along pier 1 reveals that the velocities are lowered mainly in the wake of the pile rows and in the upstream half of the pier where submergence is high (see Fig. 11). Upstream of pier 1 and 3 the flow velocities are less impacted. The most significant evolution is a flow contraction between pier 7 and 9 generating a velocity increase of approx. 0.2 m/s.

A simulation comparing flow velocities with and without the additional pier friction term has been run in order to analyse the influence of this additional friction term on the results. Comparison is presented in Fig. 12. It can be seen that the pier friction generates a decrease of flow velocities at and in the wake of pier 1, 6 and 7, leading to slightly different cross-sectional velocity profiles. The influence is strongest at pier 1 between the two pile rows where pier friction reduces the velocities by approx. -0.15 m/s (i.e. approx. 10%) while flow velocities in the centre of the river are approx. 0.05 m/s higher than the case without pier friction (flow contraction).

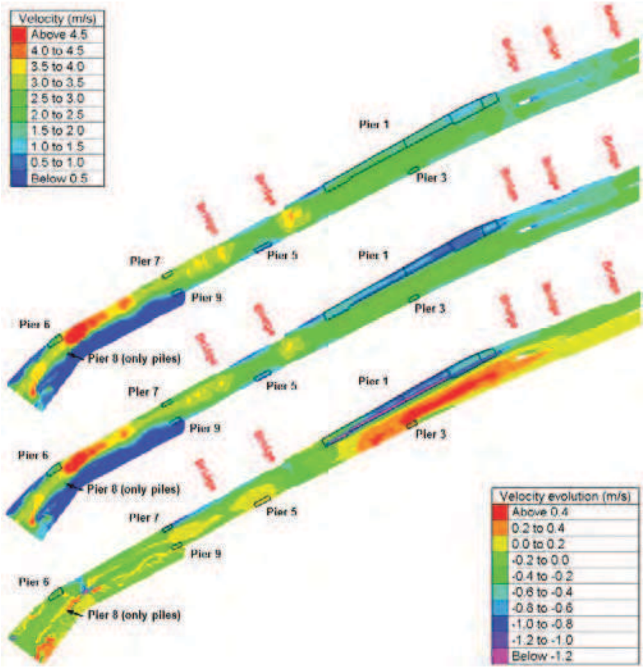

Figure 10: Project impacts on flow velocity. Top: velocities with current state geometry. Middle: velocities with project geometry. Bottom: Velocity evolutions. 100-year flood with mean sea level.

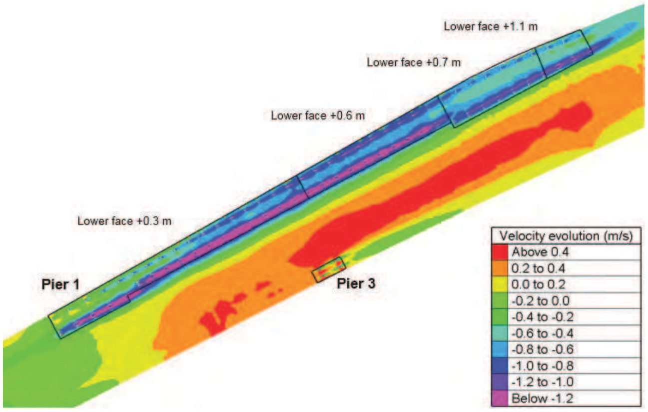

Figure 11: Detailed view of velocity evolutions along pier 1 and 3.

V. TURBULENCE-BASED EROSION RISK ASSESSMENT

A. *Accounting for turbulence in erosion processes*

Bed erosion occurs when the local bottom shear stress is greater than the material's critical shear stress. Local bottom shear stress is expressed by Eq. 5.

$$\tau = \rho \cdot u_*^2 \qquad (5)$$

Where τ is the bottom shear stress (N/m^2), ρ the water density (kg/m^3) and u_* the friction velocity (m/s). The friction velocity is calculated from the flow velocity and the bed friction coefficient. This expression is valid for flow conditions in which turbulence is generated by bottom friction. However, for flow conditions in which turbulence is also generated by other factors than bottom friction, the expression above might underestimate the actual shear stresses. For example, analysis of turbulent structures in eddies shows that

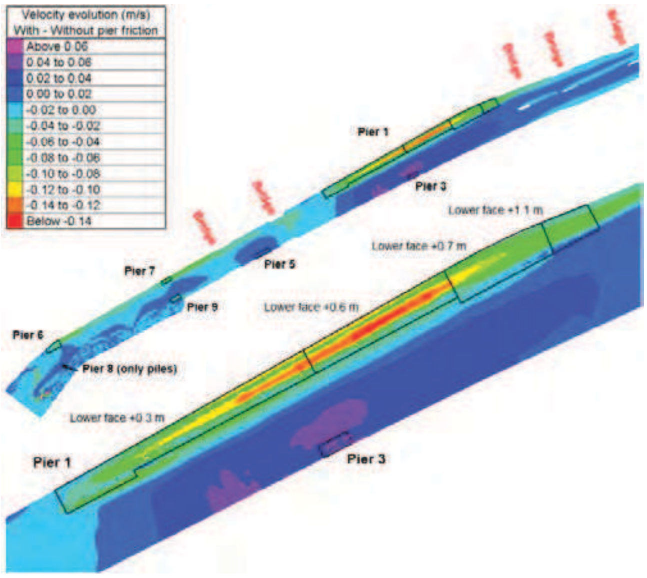

Figure 12: Velocity evolutions due to the impemented pier friction term. Top: global view. Bottom: detailed view along pier 1 and 3. Negative values indicate a decrease of velocities due to the additional pier friction term.

pressure can vary up to a factor 18 [2]. Hence it is proposed here to use a turbulence-based approach to assess the erosion risk along the Å-rummet project. This method is inspired from Hoffmans [2] and more generally from erosion protection design praxis [3].

A common way to analyse flow turbulence is to use the so-called relative turbulence intensity defined in Eq. 6.

$$r_0 = \frac{u\prime}{U} = \frac{\sqrt{k}}{U} \qquad (6)$$

Where r_0 is the relative turbulence intensity (-), u' the root mean square of the turbulent velocity fluctuations (m/s), U the local depth-averaged flow velocity and k the local depth-averaged turbulent kinematic energy (m^2/s^2). Classical values for relative turbulence intensities are presented in Table 1 [2].

TABLE 1. RELATIVE TURBULENCE INTENSITY VALUES [2]

r_0	Turbulence level	Comments
0	No turbulence	Laminar flow
< 0.08	Small turbulence	-
0.08 – 0.15	Normal turbulence	Channel, river flow
0.15 – 0.20	High turbulence	Downstream of structures (bridges, piers, etc.)
0.20 – 0.30	Very high turbulence	Downstream hydraulic jumps, sharp bends, etc.
0.30 – 0.60	Extreme turbulence	-

The turbulent kinematic energy is linked to the friction velocity by the dimensionless turbulent energy as defined in Eq. 7.

$$k^+ = \frac{k}{u_*^2} \quad (7)$$

Where k^+ is the depth-averaged dimensionless turbulent energy (-). By rearranging Eq. 6 for k, Eq. 5 and 7 can be combined to express the shear stress as a function of the relative turbulence intensity and the dimensionless turbulent energy, see Eq. 8.

$$\tau = \frac{\rho (r_0 U)^2}{k^+} \quad (8)$$

This expression can then be used to express the Shields parameter as a function of turbulence parameters, see Eq. 9.

$$\theta = \frac{\tau}{\Delta \rho g d_{50}} = \frac{1}{k^+} \cdot \frac{(r_0 U)^2}{\Delta g d_{50}} \quad (9)$$

Where θ is the Shields parameter, Δ the relative density of bottom material (typically 1.65) and d_{50} the median diameter of the riverbed material (m). In this expression, the dimensionless turbulent energy k^+ should be defined as a constant in order to keep the influence of the turbulent term $k = (r_0 U)^2$, which can be considered being valid for uniform flows. This is a weakness of this method since we introduce an uncertainty in how k^+ should be defined. In the depth-averaged k-ε model, k^+ can be assessed by Eq. 10 assuming equilibrium conditions between the turbulent energy produced by bottom friction and its dissipation rate [1].

$$k^+ = C_{2\epsilon} \cdot \frac{P_{kv}^2}{u_*^2 \cdot P_{\epsilon v}} = \left(3.6 \cdot \sqrt{C_\mu} \cdot C_f^{1/4}\right)^{-1} \quad (10)$$

Where $C_{2\epsilon}$ and C_μ are constants of the k-ε model (1.92 and 0.09 respectively) while P_{kv} and $P_{\epsilon v}$ are production terms along the vertical (see [1] for more details). k^+ values estimated by Eq. 10 are typically ranging between 2.2 and 3.5 for classical friction coefficients and flow characteristics (water depth) expected in river flow. k^+ values computed from TELEMAC-2D results using Eq. 7 in the vicinity of pier 1 and 3 in the middle of the river, that is avoiding the influence from the piers, are approximatively 2.9. This value has been chosen to assess the erosion risk.

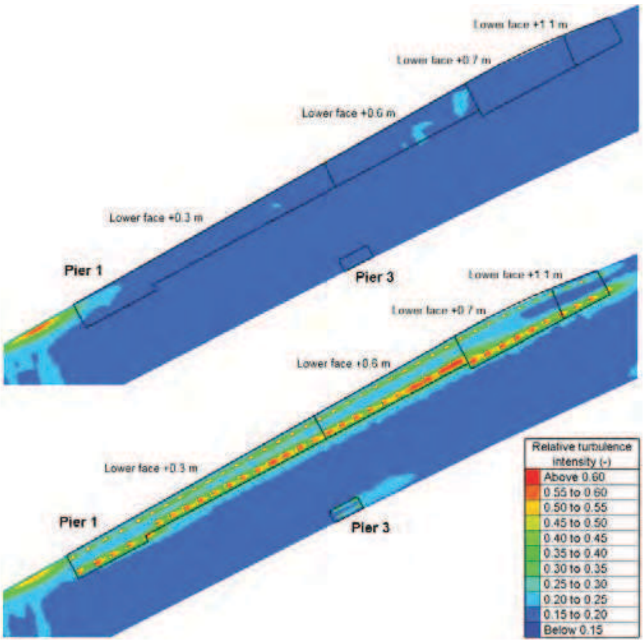

Figure 13: Relative turbulence intensities in the vicinity of pier 1 and 2. Top: current state geometry. Bottom: project geometry. 100-year flood with mean sea level.

B. Results

The relative turbulence intensities computed in the current state geometry for the 100-year flood are comprised between 0.15 and 0.20 along the planned piers which corresponds to a high turbulence level, see Fig. 13. This result is reasonable being given the flow velocities in this region (2.0 to 3.5 m/s) and the Strickler coefficient used. For the project geometry, the relative turbulence intensities increase below and in the wake of the piles. The strongest influence is observed for pier 1 where the average turbulence level is increased up to approx. 0.4 with maximum values in the wake of the outer piles, the most exposed to the current, exceeding 0.6. The turbulence level in the centre of the river is not significantly impacted.

The relative turbulence intensities were used to compute

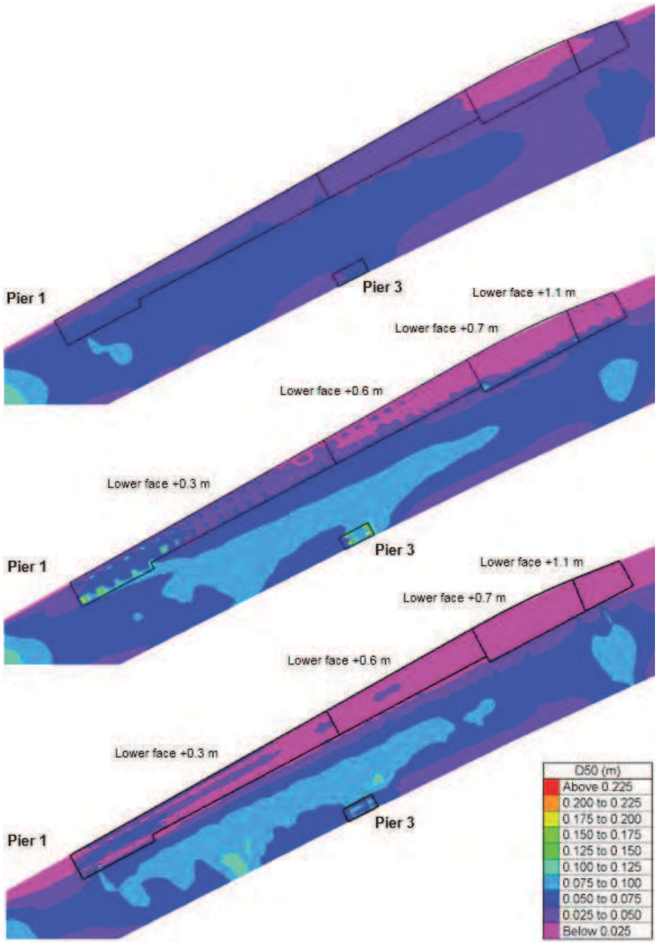

Figure 14: Critical particle size d_{50} in the vicinity of pier 1 and 3. Top: current state geometry (turbulence-based). Middle: project geometry (turbulence-based). Bottom: project geometry (classical approach). 100-year flood with mean sea level.

the critical particle size below which erosion occurs. Eq. 9 was used, rearranging for d_{50} and using the chosen dimensionless turbulent energy k^+ = 2.9. The particle Reynolds number being large (Re_* > 1000), the critical Shields parameter was chosen as θ_c = 0.06. For comparison purposes, the critical particle size has also been computed using the classical approach based on friction velocity. The obtained critical particle size is presented on Fig. 14. As expected, results show that the upstream piles of pier 1 are subject to a high erosion risk due to both high flow velocities and turbulence level. If results show that the erosion risk is high in the vicinity of the piles, it is interesting to note that the critical particle size is actually smaller in the downstream part of pier 1 compared to the current state. This is due to the fact that flow velocities are decreasing in this area compared with current state. The critical particle size also increases in the centre of the river due to the contraction effect between pier 1 and 3. It is worth noting that the classical approach clearly shows a correlation between flow velocities and erosion risk with a much lower critical particle size below and in the wake of piles than results obtained with the turbulence-based approach.

Unfortunately, no detailed information on the actual riverbed material was available for this study. Hence, this analysis has been performed mainly in order to highlight how the erosion risk is affected by the Å-rummet project. It is important to note that further analysis is required prior to using this methodology for erosion protection design, especially regarding how to define the depth-averaged dimensionless turbulent energy k^+.

VI. Conclusion

This article presents the methodology and results of a hydraulic study performed in the Gavleån River in Gävle, Sweden, in which an architectural project with piled piers within the riverbed is planned. The results showed that piers (especially pier 1), which can be submerged during high flows, induce negative impacts on water levels. However, the increase in water levels is not generating a significant aggravation of the flooding risk. Piers also induce a new cross-sectional distribution of the flow velocities with lower velocities under and in the wake of piers and higher velocities in the center of the river due to flow contraction. Flow conditions under the piers are very turbulent which has a negative impact on erosion risk.

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